APPENDIX F

RELIABILITY ANALYSIS FOR WALTER F. GEORGE LOCK AND DAM

FACT SHEET WALTER F. GEORGE LOCK, DAM AND POWERHOUSE MAJOR REHABILITATION EVALUATION REPORT PREVENTION OF STRUCTURAL FAILURE

LOCATION: Located on the Chattahoochee River, approximately 1 mile north of Fort Gaines,

Georgia, and approximately 1.6 miles upstream from the Georgia State Highway 37

bridge.

AUTHORITY: 1945 River and Harbor Act (House Document 342, 76th Congress, 1st Session), modified

by the 1946 River and Harbor Act (House Document 300, 80th (Congress, 1st Session).

PURPOSE: To provide navigation, flood control, water supply, recreation and generate electric power

on the Chattahoochee River.

TYPE OF DEFICIENCY:

The basic problem at Walter F. George involves seepage under the dam. This is a result of the deterioration of the limestone foundation material due to erosion and solutioning. Since this phenomenon cannot be accurately predicted, and distress in the structure does not become evident until the condition borders on imminent failure, this is a serious dam safety issue.

Seepage has been a problem at the Walter F. George Project since the beginning of the impoundment of the reservoir. The seepage problem can be attributed to three basic foundation conditions; (1) the presence of solution channels concentrated near the top of rock and cavernous horizons in the Earthy Limestone member of the Clayton Limestone Formation; (2) underseepage through the permeable section of the alluvial overburden; and (3) the high artesian heads in the Shell Limestone member.

HISTORY OF MAJOR SEEPAGE EVENTS:

1961: During stripping operations adjacent to the powerhouse non-overflow wall, two sinkholes developed near the dike centerline. A test excavation in the sinkhole area revealed extensive solution features. The decision was made to grout the earth dike sections on 5-foot centers to the top of the Shell Limestone.

1962: During filling of reservoir, numerous small boils developed in the drainage ditch at the downstream toe of the Alabama dike. To lessen the possibility of piping foundation material, relief wells were installed along the downstream toe of the floodplain portion of the dikes on 40-foot centers.

1968: Sinkhole activity increased on the left bank adjacent to the lock. Sinkholes occurred downstream of the dike, a spring developed adjacent to the lock wall, and a sinkhole was discovered in the lake. As a result of this sinkhole activity, remedial grouting was done along the Georgia dike, and a sand filter trench was constructed along the lock wall to intercept the seepage and prevent piping of material

1978: A supplemental design memorandum entitled "Study of Permanent Solutions to Underseepage Problems" was prepared which recommended the installation of a positive cutoff wall beneath the earth embankments to upgrade the dam to meet current standards.

A 24-inch thick concrete cutoff wall was chosen for installation to the bottom of the shell layer within the Earthy Limestone.

1981: Phase I of the cutoff wall was installed through a portion of the Georgia Dike.

1982: In between construction of the two phases of the cutoff wall, a boil was discovered immediately downstream of the powerhouse. The boil was a result of water flowing from a construction dewatering well (C-4) located at the downstream toe of the powerhouse. Construction piezometer (P-2) was found to be the source of the lake water flowing beneath the structure. The entrance point, dubbed "Hungry Hole", was plugged with a tremie pipe. The Hungry Hole and the eroded channel beneath the powerhouse was filled with 175 cubic yards of termied concrete. Additional grouting was performed along the upstream face of the powerhouse, and the upstream western one third of the spillway.

1983-1985: Phase II of the concrete cutoff wall was installed through the remainder of the Georgia Dike and the Alabama Dike.

1992-1996: Walter F. George personnel reported an increase of flow rates in piezometer SP-5 and the powerhouse drains. In response to this flow increase, searches were started in 1993 for water entrance points in the lake bottom upstream of the structures. Simultaneous with the lake searches, grouting was conducted through holes drilled through the spillway monoliths. Sixty-four holes were drilled from the top of spillway into the Shell Limestone. In June 1996, foundation material was found in the draft tube floor, beneath drain D-12. During subsequent dives, it was confirmed that foundation material had piped, and was continuing to pipe through the drain. Based on this finding, the status of the project was downgraded to "a grave maintenance condition".

FUNDS EXPENDED FOR MAJOR SEEPAGE REPAIRS:

1961: The total cost of the grouting program was \$2.3 million.

1962: The total cost of the relief well system was \$430,000.

1968: The total cost for the remedial work was \$3.2 million.

1981-1985: The cost of installing the Phase I cutoff wall was \$2.4 million, and the cost the Phase II cutoff wall was \$9.1 million.

1982: The cost of plugging "Hungry Hole" plus additional grouting was \$1.1 million.

1993-1996: Total cost for the lake searches and grouting from July 1993 through June 1996 was approximately \$2.4 million.

IMPACTS OF FAILURE:

If no action is taken to correct the seepage problem at the Walter F. George Project, it will only be a matter of time before the concrete structures are completely undermined by the river. The erosion of the limestone foundation material is expected to occur slowly, reducing the risk of loss of life. However, if the situation is left unchecked, the erosion will eventually result in the complete destruction of the spillway and powerhouse structures.

PLAN OF IMPROVEMENT:

Construction of a positive concrete cutoff wall.

RELIABILITY ANALYSIS FOR WALTER F. GEORGE LOCK AND DAM

F-1. General.

- a. A reliability analysis was performed for the overall project area within the limits of the project that will receive the recommended remedial work to be funded by the Major Rehabilitation program. The requirement for the proposed remedial measures is the result of the serious dam safety concerns for the project's integrity due to the high flows being measured from downstream sources of old piezometers, relief wells, drains, etc. Foundation materials of some size (1/2 to 1 inches) have been recovered from these discharges. Even as the known discharges are being remediated, many natural rock joints, core borings, construction piezometers, and dewatering wells located in the lake upstream of the structures and tailrace downstream of the structures may be in the process of developing into the next emergency.
- b. Reliability analysis was conducted in accordance with the general guidance obtained in Memorandum from CECW-OM dated 29 SEP 1995, titled "Guidance for Major Rehabilitation Projects for Fiscal Year 1998" and ETL 1110-2-532, "Reliability Assessment of Navigation Structures". Additional guidance, methodologies and procedures were also obtained from the following references:
- (1) Hodges Village Dam, Massachusetts, Major Rehabilitation Evaluation Report, June 1995, prepared by CENED.
- (2) Wolff, Thomas F., 26 October 1996, Letter concerning Reliability Analysis at Walter F. George under contract DACWOl-97-P-0025 for CESAM.
- (3) Wolff, Thomas F., 2 December 1996, Letter concerning review of assumed hazard function analysis under contract DACWO1-97-P-0025.
- (4) Erwin, James W., 4 January 1997, Letter concerning Expert Elicitation and Cutoff Wall Design, under contract DACWOl-97-P-0164
- (5) Erwin, James W., 29 January 1997, Letter concerning Expert Elicitation and Cutoff Wall Design, under contract DACWO I -97-P-O 164
- (6) Vick, Steven G. and Stewart, R.A., 1996 ASCE Conference on Uncertainty in Geotechnical Engineering, Risk Analysis in Dam Safety Practice., B.C. Hydro.
- (7) Von Thun, J. Lawerence, 1996 ASCE Conference on Uncertainty in Geotechnical Engineering, Risk Assessment of Nambe Falls Dam, Bureau of Reclamation.

F-2. Event Tree.

a. The reliability analysis is initiated with the preparation of an event tree with the main issue being the occurrence of significant seepage events over the course of the next fifty years. A significant seepage event is being defined as seepage from the reservoir which causes at least one of the following: a sinkhole, large increases in seepage flow, piping, or much higher than normal piezometric heads measured under and downstream of the dam. The development of a seepage event is not related to pool elevation since the greatest head differential is normal pool and tailwater conditions. The estimation of the frequency of significant seepage events 50 years into the future is based on the past history of events as they have been recorded over the life of the project and judgment as to the characteristic shape of the risk function over time. The risk of these seepage events is believed to have varied over time with an initial high risk period during and soon after construction and reservoir filling followed by a period of initially low but gradually increasing risk. The initial high risk stems from the fact that seepage problems are likely to show up upon initial exposure to high seepage gradients in any areas where existing subsurface conditions are conducive to seepage problems. The increase in risk over time after the initial high risk period is due to the progressive nature of seepage in limestone foundations, primarily due to progressive removal of fine-grained joint or cavity fill material in the limestone under high seepage gradients and to a lesser degree due to solutioning of the limestone. The deterioration of materials used for installation of relief wells and/or piezometers due to aging may also be a contributing factor. The increasing risk of seepage events over time is generally supported by the history of the seepage events at the project. Five significant seepage events have occurred over the life of the project from 1963 to date. The frequencies of the known significant seepage events, over the 33-year history of the project since reservoir filling in 1963, is shown in Table F-1. The frequencies have generally increased over time. The frequency since 1982 is significantly greater than the frequencies in prior years after the initial event in 1963. One of the events occurred immediately after reservoir filling in 1963 and the other four during the period 1982 to 1996.

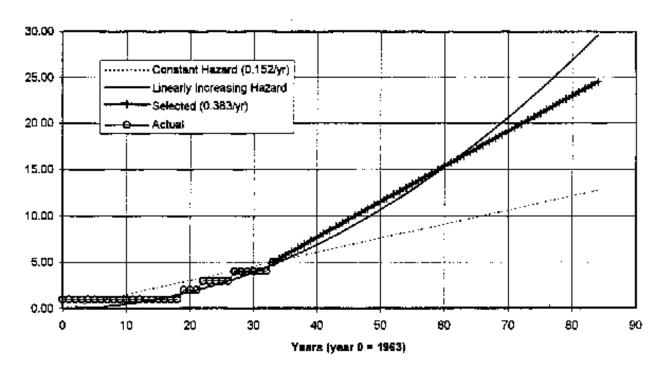
Table F-1 - Time Intervals between Significant Seepage Events

	Event	Event	Time Since	Years	Cumulative	Incremental
Event	Start	End	Reservoir	Between	Frequency	Frequency
No.	Date	Date	Filling, years	Event Starts		
1	1963	1963	0	NA	NA	NA
2	1982	1982	19	19	2/19=0.105	1/19=0.053
3	1985	1994	22	3	3/22=0.136	1/3=0.333
4	1990	1991	27	5	4/27=0.148	1/5=0.200
5	1996	1996	33	6	5/33=0.152	1/6=0.167

b. The average frequency for the 1963 to 1996 period was (5)/33=0.152 events per year. To estimate the frequency of significant seepage events 50 years into the future Figure F-1 was used. In Figure F-1 the actual seepage event are plotted as circles extended over the

time period to the next seepage event. For the initial time period 1963 to 1996 a constant hazard function line of $\lambda=0.152$ is plotted. The best fit to these initial seepage events is obtained using a Weibull distribution with a linearly increasing hazard function (see Figure F-1). However for computational ease it was decided to use a constant hazard function during the fifty year period from 1997 to 2047 instead of the linearly increasing hazard function. This constant hazard function line was determined by drawing a straight line for the time period of 34 to 84 years on the plot in Figure F-1, which closely approximates the curved linearly increasing hazard function line. This straight line is shown on Figure F-1 as the selected (0.383/yr) line. This straight line is a constant hazard function line with $\lambda=0.383$. The value of λ which is the slope of the straight line was obtained as follows:

$$\lambda = \frac{SeepageEvents}{Years} = \frac{(24.2 - 5) events}{(84 - 34) years} = 0.383$$



Number of Seepage Events Figure F-1

c. The hazard function described above and presented in Figure F-1 is based upon the number of known significant seepage events in the past and provides an estimate of the expected number of known events in the future. The event tree includes some unknown events that would not have been counted as significant seepage events in the past if they have occurred. The appropriate frequency to be used with the initial event Al of the event tree should be the frequency of all events considered in the event tree including unknown seepage

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events. The frequency of unknown events is estimated based on back-analysis of the probabilities assigned by expert opinion to the events with "no problem" consequences most likely to be unknown. Specifically the probability of an event being unknown is estimated to be approximately the same as the probability of event C2 of the event tree. The probability of event C2 given that a significant event has occurred is the probability of the seepage being well connected to the reservoir only combined with the probability of the source being located far from structures. Therefore the probability of an event being unknown is estimated as 0.0880 based on the expert elicitation process and the probability of an event being known is estimated as the complement 0.9120. The frequency of unknown events is estimated as $(0.0880/0.9120) \times 0.383 = 0.037$ events per year for the 1997-2047 period. The total frequency of known and unknown significant seepage events used for the initial event Al of the event tree is estimated as 0.383+0.037 = 0.420 events per year over the 1997-2047 period. The expected number of known and unknown events over the 1997-2047 period is estimated as 0.420x50 = 21.0 events for the same period.

d. Poisson Distribution on the Number of Possible Events in a Time Period t. The hazard function value h(t) = 0.420 events per year indicates that each year, there is a constant probability of p = 0.42 that an event will occur. Hence, over a long period of t years, the expected number of events N is

$$E[n] = tp = 0.42 n$$

For a fifty-year period, the expected number of events is 21.

However, consider the fact that the number of events in a 50-year period will not always be exactly 21, as the process is random. In some periods there might be 23, 18, etc. Over a long time with n approaching infinity, the number of events would approach 0.42N.

When the hazard function is constant, events are assumed to occur at an average rate that is constant with time. In this case the probability distribution on the number of events N in the time period t is defined by the Poisson distribution, a discrete distribution given by

$$Pr\left[\left(n=x\right) \mid t\right] \ = \ \frac{\left(\lambda t\right)^{x}e^{-\lambda t}}{x!}$$

where

Pr[(n = x) | t) is the probability that the number of events N in time period t will equal the value x

λ is the average number of events per time, which can be estimated from historical data, and

e is the base of the natural logarithms

Substituting the values x = 1, 2, ... in the above equation, with $\lambda = 0.42$ and t = 50, the probability values in Table F-2 are obtained. These give the probability distribution on the number of seepage events in a 50-year period. Values less than 4 and above 41 have negligible probabilities.

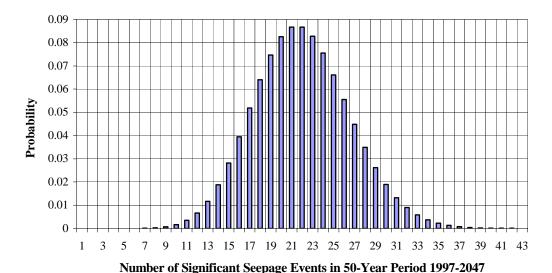
The probability that the number of events is between two values can be determined by summing the probabilities associated with the corresponding values. For example, the probability that the number of actual events in a 50-year interval is between 17 and 24 is

$$Pr (17 \le n \le 24) = 0.06404 + 0.07472 + ... + 0.06613$$
$$= 0.6192$$

which is more likely than not.

Table F-2. Probability of n Significant Seepage Events in 50-Year Period 1997 – 2047

No. of	Pr	No. of	Pr	No. of	Pr
events, n		events, n	11	events, n	
0	7.58E-10	15	0.0395	30	0.01327
1	1.59E-08	16	0.05184	31	0.00899
2	1.67E-07	17	0.06404	32	0.0059
3	1.17E-06	18	0.07472	33	0.00375
4	0.00001	19	0.08258	34	0.00232
5	0.00003	20	0.08671	35	0.00139
6	0.00009	21	0.08671	36	0.00081
7	0.00027	22	0.08277	37	0.00046
8	0.00071	23	0.07557	38	0.00025
9	0.00166	24	0.06613	39	0.00014
10	0.00349	25	0.05555	40	0.00007
11	0.00665	26	0.04486	41	0.00004
12	0.01164	27	0.03489	42	1.84E-05
13	0.01881	28	0.02617		
14	0.02821	29	0.01895		



Probability of n Significant Seepage Events in 50-Year Period 1997-2047 Figure F-2

- e. The range of significant events to occur goes from 4 to 41 with probabilities occurring outside that range being so small they can be considered negligible. The number of events with the highest probability of occurrence is 21, as shown on Figure F-2. One should bear in mind that this number is the total number of events within the limits of the reservoir. The reservoir limit is deemed to be approximately 8,000 feet upstream of the existing dam and runs the entire width of the concrete monoliths in place (2000 feet±). Approximately 8,000 feet is the distance where the earthy limestone is discontinuous and the pool can enter into the Shelly Limestone directly.
- F-3. <u>Seepage Event Characteristics</u>. Once a seepage event has occurred, then one must ask what characteristics of the event can manifest itself. The two main characteristics that will determine the consequences of the seepage event is its connection to headwater only, connection to both headwater and tailwater, or tailwater only and the distance that the source occurs from the structures.
- a. A seepage connection to headwater only implies that there is no direct connection for flow to exit downstream of any structure but the full uplift pressure could be felt at the base of a monolith for most if not all of its base contact area. Seepage connected to both headwater and tailwater assumes that the reservoir head has a direct path from some area upstream of the dam, down to some strata below the bearing elevation of the structure, travel horizontally for its full length and exit some distance downstream. In classic seepage analysis, this exit point is next to the toe of the structure at the surface, but since the seepage media is channelized limestone, the exit could be some distance downstream without significant head loss. The final seepage connection would be to tailwater only.

This connection has no consequence for failure since is does not threaten the performance of the structures as defined herein.

- b. The distance that the source occurs from the dam is defined by Near (0 to 300 feet) and Far (300 to 8000 feet). The 300 feet criteria was determined by the distance that barges could be placed against the dam and receive support for grouting during emergency repairs and the likelihood that a sinkhole would be noticed and the path length for seepage calculations. Far was considered for the 300 foot distance to the 8000 foot area in the reservoir where the impervious Earthy Limestone is discontinuous and the pool feeds directly into the Shelly Limestone.
- F-4. <u>Seepage Consequences</u>. Once the characteristics of the seepage event have been determined, then some action called emergency repairs is required that will generate a cost or the event is unknown or located where repair action can not readily deal with it and the consequences will be evident later. These emergency repair actions are classified as successful, only partly successful, unsuccessful or not undertaken.
- a. Successful is defined by repair action that fills the sinkhole and/or returns the observed flows and piezometric heads back to the status that was present prior to the event.
- b. Partly successful will be defined as repair efforts that only remand a portion of the emergency situation back to its prior state. One example of this would be grouting of a sinkhole that only partially reduced measured flows or piezometric heads that remained above prior levels but the sinkhole was filled.
- c. An unsuccessful repair is where the action has virtually no effect on the flow rates or the piezometric heads. An example of this would be grouting (chemical or cementitious) where the grout consistency was so thin and/or the seepage velocities so high that the grout merely washed out downstream or the grout set up so quickly that it plugged only the initial section of the opening.
- d. Emergency repairs not taken means the seepage source was not discovered or surfaced too late for any repairs to be performed.
- F-5. <u>Emergency Repair Consequences</u>. The direct result of the various unsuccessful emergency repair actions or lack of emergency repair action will be a major structure failure, a minor structure failure, or no problem for headwater only connected seepage events and excessive piping, excessive settlement beneath a monolith or no problems are observed for a seepage event connected to headwater and tailwater.
- a. Major structural failure is defined as sufficient movement of a monolith to prevent it from performing its intended function. This movement could take the form of sliding or overturning from high uplift pressures to the degree that pool could not be maintained or power could not be generated or the lock could not be operated.
- b. Minor structural failure is defined as relatively small movements of a monolith after which the functionality of the structure is retained but some repair is likely. An example of this would be a binding of a spillway gate.

- c. Excessive piping is defined as flow measured from downstream sources that cannot be controlled or can only be reduced to a level, which still produces the transport of foundation materials from erosion. The path may be small or well below the foundation subgrade of the monolith and a loss of bearing area is not critical.
- d. Excessive structural settlement or rotation will result from a cavity of sufficient size on the downstream end to cause loss of contact area back to the resultant. Based on the structural analysis, the monolith will remain stable until that happens, thereby producing a major structural failure without warning.
- F-6. <u>Results of Unsatisfactory Performance</u>. All of these consequences result in varying degrees of unsatisfactory performance, which induce some amount of cost. These unsatisfactory performance events include a catastrophic dam failure, repairs to structure with a controlled drawdown of pool, or repairs to the structures without a pool drawdown being required.
- a. Catastrophic dam failure is defined as uncontrolled flows resulting in a drawdown of the reservoir in less than 11 days. This criteria stems from the hydraulic analysis where for a given inflow, the complete drawdown of the pool must occur through the turbine draft tubes and the higher discharges will severely damage the turbines and some high erosional velocities will be felt on downstream structures.
- b. The other events consist of repairs that require the pool to be drawn down but can wait until after 11 days or the pool is lost by a failure but the rate of loss is slow.
- c. The third performance event is where repairs can be made without requiring the pool to be drawn down. These could be repairs similar to the emergency repairs in the pool or drilling from the top of dam.
- F-7. Economic Consequences of an Unsatisfactory Performance Event. The following repair actions for economic analysis assume that the structure in question is a spillway monolith, which should provide an average level of effort between repairs to a lock monolith and repairs to a powerhouse monolith. This is not conservative since it is likely that more than one monolith could be involved and the costs would be increased accordingly.
- a. Major Structure Failure Catastrophic Dam Failure. This situation assumes that the monolith has moved, cracked and /or the gate which is in the opened position cannot be operated. Pool is being lost at a rate greater than 11 days. There will be turbine damage if the pool is intentionally drawn down at this rate. In order to effect repairs, a design must be prepared and plans and specifications produced for bid. This should take about three months to complete assuming O&M is used for funding. For construction, a cofferdam must be constructed around the upstream and downstream ends. With a 50-foot monolith width, assume three cells are required upstream and downstream to effectively dewater the work area. A total of six cells, assuming one month per cell to install, will take six months. Each cell costs some \$200,000 to install with some \$200,000 needed for mobilization and template

construction. The existing monolith must be chipped out and the cavity in the foundation must be treated. Assume this takes two months to accomplish and costs \$150,000. The 130-foot monolith must be placed in 5-foot lifts. This means 26 lifts are required, and at one week per lift, will take six months to perform. The total concrete volume is 18,648 cubic yards. Assuming \$150 per cubic yard for placement costs, this portion of the work will cost some \$3,000,000. The new gate must be set and the new driveway bridge erected and poured. This should take an additional three months to complete and cost about \$600,000. The turbine blades will be reworked due to cavitation damage. This work will take six months and cost \$200,000. Assuming that money is immediately available, a total of about 20 months is required to bring the structure back to pre-event condition. The total cost to accomplish this work is about \$5,200,000. Approximately five months will be needed to refill the reservoir to pre-loss conditions based on normal flows. The pool could start to be refilled after the cofferdam is in place.

- b. Major Structure Failure- Repair Structure with Controlled Pool Drawdown. This situation will require the same time line as that above except that due to the slow release of pool, the turbines will not be damaged and no damaging velocities will be felt downstream. The repair procedure and time should be the same as well as the cost less that accounted for the turbine repairs. This should add up to \$5,000,000.
- c. Minor Structure Failure-Repair with Pool Drawdown. This condition implies that the failure did not result in the immediate loss of pool and that the damage to the monolith is not severe enough to require replacement. The damage is severe enough to require chemical grouting and the velocities flowing through the cracks are too great for success. A controlled pool drawdown is required to effect repairs but the level is sufficiently low to prevent navigation, power generation, flood control and some recreation. Due to the numerous obstructions within the monolith, the grouting will take six months to accomplish using one crew full time and using an on site grout plant. The cost should add to about \$750,000.00 based on out recent historical grouting efforts.
- d. Minor Structure Failure- Repair without Pool Drawdown. This condition will be similar to the emergency repair efforts in terms of overall time but the costs should be somewhat less due to less problems with the grout mix design. Flood control may be affected, depending on the time of year of the event. We are assuming that one drill crew will be dedicated full time for five to six months. The overall costs for this work should be \$500,000.
- e. Emergency Repairs Prior to a Failure. This condition includes the remedial repairs to a discovered sinkhole found in the reservoir or known cavities beneath the structure that can be reached by drilling through the structure from the top driveway or from a barge anchored immediately upstream. The pool is not drawn down but the weather must not produce a severe fetch. We are assuming the average repair will take six months to perform using one five-person drill crew, the use of the government owned barge and support boat. The cost should run about \$500,000.

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f. No Problem. This condition is described as a result of the emergency repairs being only partly successful or unsuccessful and no immediate consequence is observed. This may be an intermediate condition where the emergency repairs are reinstated at a later date, perhaps after a potential upstream source is identified from soundings in the reservoir or observations from a diver.

F-8. Expert Elicitation.

- a. General. The reliability of a structure shall be stated in terms of the probability of unsatisfactory performance of the feature. Unsatisfactory performance of a component may be indicated at various levels of performance, depending on the consequences of that performance level, from minor deflections to complete collapse of a structure. The guidance mandated that the probabilities of unsatisfactory performance be calculated using one of four methods: (I) Reliability Indices; (2) Hazard Functions; (3) Historical Frequency of Occurrence Analyses and (4) Expert Elicitation. Some of the above methods rely on a frequency approach requiring a significant database of historical events to judge the performance of a structure in the future. This data can take the form of the number of dam failures, the application of flood elevations to a reduced factor of safety, such as slope stability or conventional seepage through an earth embankment. In these cases, where classical failure mechanisms (sliding, overturning, structural deterioration, etc.) can be applied with confidence, then statistical analysis is warranted. Geotechnical engineering in general and karst bearing formations in particular do not lend themselves to the above. There are very few events to base any statistical analysis and the performance of the formations over time are very site specific, especially where normal operating conditions are the driving forces for degradation. Dam safety risk assessment can not be performed without liberal application of engineering judgment, which is subjective using expert opinion with a great deal of experience with the performance of the formation in question or similar materials elsewhere. The method is highly dependent on the experience and skill of the panel of experts selected and the procedures used to avoid biases in the probabilities. In consultation with CECW-ED and Dr. Wolff of Michigan State University, this office established the following procedures.
- b. Board of Consultants. A Board of Consultants was convened on 21-22 May 1996 to review the underseepage problem experienced at Walter F. George Reservoir and to recommend potential solutions to the problem. The Board Members had many years of experience in geology and geotechnical engineering issues of major civil type structures in the United States as well as abroad, with representation from academia, engineering design, construction, and government. After much "brainstorming" of the issues, the group listed the main causes of seepage as: (1) inherent discontinuities in the Earthy Limestone formation from joints, natural weakness, sinkholes, etc; (2) manmade penetrations, such as core borings, monitoring and dewatering wells, sheetpile penetrations, and excavations for foundations; (3) erosion and solutioning of the formation due to seepage velocities and the chemical determination of the water.

The Board reinstituted the brainstorming session for potential solutions to the identified seepage causes. The group listed five possible courses of action that the District could undertake for the future of Walter F. George: (1) Do Nothing; (2) Continue current activities with grouting; (3) Install an impervious blanket upstream of the dam; (4) Perform innovative grouting; and (5) Install a concrete cutoff wall with a concrete apron immediately upstream of the dam structures. The only alternative that is considered capable of actually cutting off the seepage from upstream to downstream without a high risk of voids is the concrete cutoff wall. It is the District's position that the concrete cutoff wall is the only remedial measure that will accomplish the task without actually inducing other mechanisms for failure. Note: The NED plan must be an alternative that is feasible and reliable. Alternatives that are not feasible and reliable should be screened out early in the process and not carried to a conclusion. A brief write up discussing alternatives that are not considered feasible and reliable will suffice.

- c. Expert Panel Member Selection. The District was then tasked to prepare a Major Rehabilitation Evaluation Report for Walter F. George Dam, using the recommendations of the Board of Consultants as guidance for remediation techniques. The first task of the report team was to prepare an event tree that describes the problems and resulting consequences. This was accomplished with input from geotechnical, geological, structural, hydraulic and hydrologic and economic disciplines. The event tree was created based on the "without project" or base condition and duplicated for the "with project (Base of wall at El. 20)" and the "with project (Base at El. -3.0 to -5.0)" condition. The use of statistical methods based on historical events could only be performed on the initial tree event. Probabilities for the subsequent events of the tree were based on engineering judgment using expert elicitation techniques since this was deemed to be the most applicable to the issue of seepage through limestone formations. The District assembled a team of geologists with many years of experience with geologic formations in general and with the behavior of the formations beneath Walter F. George in particular. This procedure was discussed with the technical counterparts in HQ and approval was given for this approach. It was suggested and agreed that an independent source outside of Mobile District be used in addition to the in-house personnel to give additional guidance and credence to the process. It was the District's belief that a renowned geologist in the private sector would give the impression of "less bias" but would be disadvantaged in predicting the behavior of formations in which he/she had no direct experience. The District desired a source of national stature that had not worked directly for Mobile District but who had some experience with the seepage events at this location. Dr. Jim Erwin, retired Division Geologist from the South Atlantic Division, fit all these requirements and agreed to serve in this capacity.
- d. Procedures for Expert Elicitation. Expert elicitation is the use of highly experienced technical judgment to establish subjective probabilities to measure an individual's degree of belief concerning the likelihood of the occurrence of an event. Subjective probabilities are generally used whenever there is insufficient data to develop the probability of an event from historical frequency of occurrence or to conduct an analytical assessment of the probability. The method is highly dependent upon the experience and skill of the panel of experts selected and the procedures used to avoid biases in the probabilities. The procedure is

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primarily used for events that have a probability of occurrence between 0.001 and 0.999 since rare events with very low probabilities are more vulnerable to bias. Consistent methods must be used to reduce this bias.

- (1) After each panel member was briefed on the project and the objectives of the Major Rehabilitation Report, they were asked to identify the most likely probability of a sub-event to occur as they proceeded through the event tree. They went back through the tree and established the range with upper and lower limits of probability for each given sub-event. The main activity of the event tree is a seepage event, which is defined as seepage from the reservoir which causes one or more of the following: a sinkhole; a large increase in seepage flow; piping of foundation material; or significantly higher than normal piezometric heads. These probabilities will have upper and lower limits of 0.999 to 0.001 and with all sub-events within that portion of the tree adding up to 1.0. The vote was given to the moderator by closed ballot for review to see if all panel members understand the event. If any analysis looked to be severely out of order, the moderator asked the respondent to reconsider the probability of the sub-event and ask any clarifying questions. Once all reasonable probabilities were returned, the probabilities for each sub event were averaged between the panel members and reported as the average most likely for that sub-event. The panel's high and low range of probabilities for that sub-event was averaged also. The final results of these probabilities were used in the calculation of risk assessment and economic consequence of that risk. This procedure was repeated through the entire event tree for the without project, with project (Base at El. 20 feet, NGVD) and the recommended plan, with project (Base at El. -3.0 to -5.0 feet, NGVD) conditions.
- (2) The product of the most likely probabilities for each sub-event as it progressed throughout the event tree was calculated and these results have been incorporated into the economic analysis calculations.
- e. With Project Probability of a Seepage Event. Since there is no historical data concerning the future number of likely events with the wall in place, the group was asked to predict the number of events likely to occur over the next 50 years given the reduction in pool area with possible joints and penetrations to act as sources of seepage. Estimates were given for the wall based at El. 20 and repeated for the wall extending down to El. –3.0 to -5.0 feet, NGVD. These numbers were totaled and averaged and then divided by 50 to give the average number of events per year. See Table F-3. These average frequencies were used below to estimate the probability of a given number of significant events over the 50-year period.

Table F-3. Predicted Number of Future Seepage Events for next the 50 Years with the Walls in Place

							Average
Plan	Dr.	John	Ross	James	Juan		per
El.	Erwin	Baehr	McCollum	Sanders	Payne	Average	Annum
20	6	2	3	1	2	2.8	0.056
-3 to -5	0	0	1	1	0	0.4	0.008

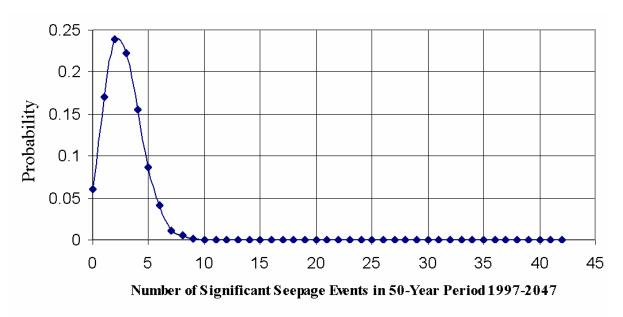
The estimated probability of the total number of significant seepage events over the period 1997-2047 are shown in Table F-4 and illustrated in Figure F-3 for the wall based at El. 20. This probability is based on the Poisson equation:

$$Pr = \frac{(\lambda t)^n e^{-\lambda t}}{n!}$$
 where, Pr is the probability of n events occurring in time interval t (50 years)

and 2 is the frequency (0.056 events per year). The process was repeated with the bottom of the wall set 3 feet into the black clay stratum of the Providence Sand Formation (El. -3.0 to -5.0 feet, NGVD) and a frequency of 0.008 events per year was used to calculate the range of probabilities shown in Table F-5 and Figure F-4.

Table F-4. (Bottom of Wall at EL. 20) Probability of n Significant Seepage Events in 50-Year Period 1997-2047

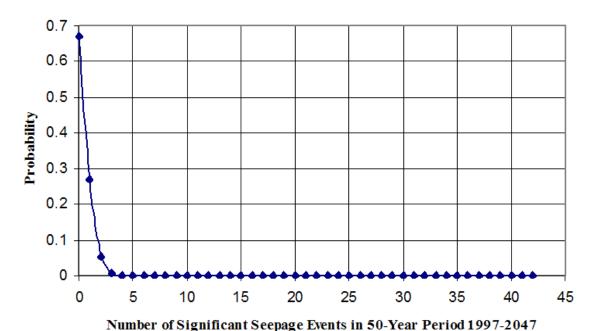
No. of	Pr	No. of	Pr	No. of	Pr
events,		events,		events,	
n		n		n	
0	0.06081	15	0.00000	30	0.00000
1	0.170268	16	0.00000	31	0.00000
2	0.238375	17	0.00000	32	0.00000
3	0.222484	18	0.00000	33	0.00000
4	0.15574	19	0.00000	34	0.00000
5	0.08721	20	0.00000	35	0.00000
6	0.0407	21	0.00000	36	0.00000
7	0.01628	22	0.00000	37	0.00000
8	0.0057	23	0.00000	38	0.00000
9	0.00177	24	0.00000	39	0.00000
10	0.0005	25	0.00000	40	0.00000
11	0.00013	26	0.00000	41	0.00000
12	0.00003	27	0.00000	42	2.61E-34
13	0.00001	28	0.00000		
14	0.00000	29	0.00000		



Probability of n Significant Seepage Events in Period 1997-2047 (Bottom of Wall at EL. 20) Figure F-3

Table F-5. (Bottom of Wall at El. -3.0 to -5.0) Probability of n Significant Seepage Events in 50 Year Period 1997-2047

		1			
No. of	Pr	No. of	Pr	No. of	Pr
events, n		events, n		events, n	
0	0.67032	15	0.00000	30	0.00000
1	0.268128	16	0.00000	31	0.00000
2	0.053626	17	0.00000	32	0.00000
3	0.00715	18	0.00000	33	0.00000
4	0.00072	19	0.00000	34	0.00000
5	0.00006	20	0.00000	35	0.00000
6	0.000001	21	0.00000	36	0.00000
7	0.00000	22	0.00000	37	0.00000
8	0.00000	23	0.00000	38	0.00000
9	0.00000	24	0.00000	39	0.00000
10	0.00000	25	0.00000	40	0.00000
11	0.00000	26	0.00000	41	0.00000
12	0.00000	27	0.00000	42	9.23E-69
13	0.00000	28	0.00000		
14	0.00000	29	0.00000		



Probability of n Significant Seepage Events in 50-Year Period 1997-2047 (Bottom of Wall at El. -3.0 to -5.0)

Figure F-4.

F-9. Conclusions. This report has summarized the approach and results of the reliability analysis to assess the adequacy of the concrete structures against potentially erosive seepage flows from an increasing number of seepage incidents similar to those which have already occurred in significant numbers. These limestone foundations are the issue of the Evaluation and will be rehabilitated by the Recommended Plan. Pool levels are not an issue since normal pool creates the most differential head for seepage. The efficiency of the structure is not an issue since the monoliths have been designed for full uplift head for sliding and overturning and the recommended plan does not touch the structures. Therefore, all evaluations and rehabilitations have considered the limestone formations alone. Geotechnical features are rather unique for evaluation since there are few historical events and performance is very site specific. Significant engineering judgment from many years of experience has always been the hallmark of geotechnical engineering and probably always will be. It is our firm belief that use of highly experienced technical judgment by expert elicitation is the only way to analyze risk of future performance for the specific geotechnical events under consideration, i.e. erosive seepage in limestone, and the ability to locate and repair the seeps. We have known that Headquarters realized this when the criteria allowed the use of this technique and it is Mobile District's firm belief that it's guidance used by the experts produced the most technically feasible results with the least bias. This belief was facilitated by the development of a comprehensive event tree, which chronicled each cause and effect relationship to its logical conclusion and established costs for each action. These methods have been discussed and approved by technical disciplines in Headquarters and we trust what we have done here can be used and improved for the next Major Rehabilitation Report with similar issues.